Ground Vibration at Seventh Street in Rochester due to DM&E Trains

[THIS PAGE INTENTIONALLY LEFT BLANK]

WILSON, IHRIG & ASSOCIATES, INC.
ACOUSTICAL CONSULTANTS

5776 BROADWAY
OAKLAND, CA
U.S.A. 94618-1531
Tel: (510) 658-6719
Fax: (510) 652-4441
E-mail: info@wiai.com
Web: www.wiai.com

GROUND VIBRATION AT SEVENTH STREET IN ROCHESTER DUE TO DM&E TRAINS

22 August 2001

Submitted to:

Mr. Stephen Thornhill
Burns & McDonnell
Kansas City, Missouri

Prepared by:

Dr. James T. Nelson
Wilson, Ihrig & Associates, Inc.
Oakland, California

INTRODUCTION

The DM&E Railroad proposes to upgrade track and introduce unit coal trains from the Powder River Basin through Rochester, Minnesota. The upgrade would include replacement or addition of track with continuous welded rail. Train speeds would be increased from the current 15 mph to approximately 45 mph.

Measurements were conducted on June 6, 2001 at the Seventh Avenue grade crossing at distances of 50, 100, 150, and 250 feet from the track to determine the potential for ground vibration impact on residential structures and habitation, and to provide supporting data for assessment of vibration impacts at other sites such as the Mayo Clinic and the Federal Medical Facility. (Measurements were conducted at the Mayo Clinic Charlton North building on a separate date, and reported previously in a report dated August 14, 2001.)

PROCEEDURE

The data collected at the Seventh Avenue NW grade crossing were obtained with piezo-electric accelerometers mounted on the side walk at 50, 100, 150, and 250 feet from the track. This site should be representative of other locations in the City of Rochester at the south side of the railroad alignment, including the Mayo Clinic area, where there is relatively shallow rock base, though a soils report was not obtained for the specific test site. The data were collected for a single train passby, and recorded on digital magnetic tape for laboratory analysis. The train speed during passage was approximately 10 mph.

The laboratory analysis included reproduction of time domain waveforms to assess peak particle velocities, peak particle accelerations, and 1/3 octave band velocities. The peak particle velocities may be compared with typical building damage criteria for transportation sources of plus or minus 0.2 in/second, though these criteria may range from 0.1 to 2 inch/second. Also, the peak particle acceleration data are often compared with vibration specifications for sensitive equipment. The 1/3 octave vibration velocity data are comparable with criteria for human exposure to vibration in buildings, such as ANSI Standard S3.29. They may also be compared with generic criteria for sensitive laboratory equipment.

RESULTS

Ground surface velocity data versus time are presented in Figure 1 for distances of 50, 100, 150, and 250 feet from the track. The scales at the right hand side refer to the span of the chart. In this case, the span is 0.02 inch/sec, so that the plus and minus full scales would be plus and minus 0.01 in/sec, respectively. The entire 360 seconds of recorded velocity data are represented for a bandwidth of approximately 1 Hz to 200 Hz. These data were obtained by reading the data file from the digital tape record, pre-filtering, and decimating the data by a factor of 24 to reduce the data size. The data are generally within an amplitude of 0.01 in/sec, an order of magnitude less than typical structural damage criteria.

Ground surface acceleration data are presented in Figure 2. These data were also obtained by pre-filtering and decimation of the recorded acceleration data. The scales indicated at the left hand sides of each of the graphs refer to the span of each of the graphs, which are 0.02 g. The full scales are thus plus and minus 0.01 g. At a range of 250 feet, the peak acceleration is of the order of 0.001 g. One may expect lower magnitudes of acceleration at greater distances.

One third octave vibration velocity levels for the train passby are plotted in Figure 3 through Figure 6. Figure 3 displays data that were obtained during passage of the two locomotives. The remaining figures were obtained from three separate samples during the course of the passage.

Also plotted in each of the figures of 1/3 octave data are criteria for residential structures and various laboratory equipment. The residential criterion curve has been applied for many years to transit system vibration, and would be applicable to ground vibration from trains. The residential criterion is recommended by the National Institute of Standards and Technology (ANSI S3.29), and is consistent with criteria for overall vibration velocity published by the Federal Transit Administration for transit systems, though the FTA relaxes the criterion for infrequent train events such as railroad train passby's. The level of 72 dB corresponds to a root-mean-square vibration velocity magnitude of 4,000 micro-in/second, or 0.004 in/second. The FTA also recommends a limit of 2,000 micro-in/second for floor vibration, corresponding to 66 dB, at vibration sensitive facilities, which is roughly consistent with criteria for bench microscopes of magnification less than 400x.

The measured ground vibration levels for the train at 50 feet from the track are within the residential limits and the criterion curve for bench micro-scopes of magnification less than 400x. At 100 feet and greater distances, the vibration levels are within all of the criteria shown. The overall vibration velocity levels indicated at the left hand side of each of the graphs are all within the FTA recommendations for ground borne vibration in either residences or vibration sensitive facilities.

The attenuation between 50 and 100 feet is relatively high for frequencies between 20 and 63 Hz. The peak in the spectrum occurs at about 25 to 40 Hz, and is likely a resonance condition of the upper soil layer over the underlying rock. Beyond 100 feet from the track, the attenuation rate is low, consistent with vibration propagation in the underlying rock strata, which probably exhibits low damping properties.

DISCUSSION

The observed ground vibration velocities suggest that homes founded on soils of similar type to those soils existing at this measurement location are very likely not being damaged by vibration caused by train operations. Increasing the train speed to 45 mph from 10 mph would tend to increase vibration levels. A typical assumption is that the vibration velocity magnitude would increase by 6 dB per doubling of train speed, which is equivalent to assuming that the vibration amplitude would increase linearly with train speed. Thus, if peak particle vibration velocities are currently of the order of 0.01 in/second, the peak particle velocity for 45 mph trains would likely be of the order of 0.04 in/second, still well within damage criterion of 0.2 in/second ppv for structures. The introduction of upgraded track with continuous welded rail (CWR) would tend to reduce ground vibration levels, so that peak particle velocities during passage of unit coal trains at 45 mph on upgraded track would likely be less than 0.04 in/second, provided that the rail is without undulation or imperfections, and that the wheels are well maintained.

The 1/3 octave band velocity levels would likely increase by approximately 12 dB, consistent a 6 dB per doubling of train speed assumption. Again, introduction continuous welded rail would tend to reduce vibration. The typical assumption is that vibration levels are reduced by about 5 to 10 decibels with continuous welded rail, but this depends on rail straightness and wheel condition. Thus, the third octave vibration velocity levels will likely increase by approximately 6 decibels, raising the overall vibration velocity level at 50 feet from the track from about 66 dB (0.002 in/second rms) to about 72 dB (0.004 in/second rms). At greater distances, the vibration velocity levels would be lower, perhaps of the order of 64 dB (0.0018 in/second rms), and well within the criterion for residential buildings.

The residential buildings may amplify vibration relative to free surface ground vibration levels. The amplification depends on the vibration coupling loss or attenuation that occurs as the vibration transmits from the ground into the foundation, and the amplification by the floor due to resonances of the floor. The amplification is dependent on the mass, stiffness, and damping of the floors. For slab-on-grade floors, the amplification is nil. For wood frame floors, the amplification could be as high as five decibels, though the foundation coupling loss may limit the total building response to a few decibels. A reasonable estimate would be three decibels for building response, which would raise the overall floor vibration velocity at 50 feet from the track to about 75 dB, midway between the FTA criteria of 72 and 78 dB for frequent and infrequent events. At 100 feet, the vibration levels would be within the 72 dB criterion for frequent events. The maximum 1/3 octave band levels at 50 feet from the track were about 62 dB during locomotive passage. Adding the same 9 dB increment to account for increased train speed, continuous welded rail, and building response, would suggest that the maximum 1/3 octave floor vibration levels would be about 69 dB, within the criterion curve plotted on each of the 1/3 octave band charts.

Thus, at 100 feet and beyond, there would be little risk of adverse community reaction to ground borne vibration from trains, and between 50 and 100 feet there might be some adverse reaction, though the vibration might still be judged acceptable for human habitation. Judging from the rapidity of attenuation with distance between 50 and 100 feet, the vibration levels at 75 feet from the track would likely be within the 72 dB FTA criterion as well. Pathological conditions could produce vibration levels higher than those suggested here, due to under-damped structural resonances, rough track, flatted wheels, and unique ground conditions.

The shallow rock base that may be typical of Rochester at the south side of the railroad is likely controlling ground vibration to low levels. Without this base, vibration levels would likely be much higher, possibly of the order of 0.1 in/second. The magnitude of ground vibration is strongly controlled by the stiffness of the soil and base rock. The shallow layer of soil tends to concentrate ground vibration near the surface. In spite of this, the magnitudes are well within criteria for residences and most other types of activities at relatively short distances from the track.

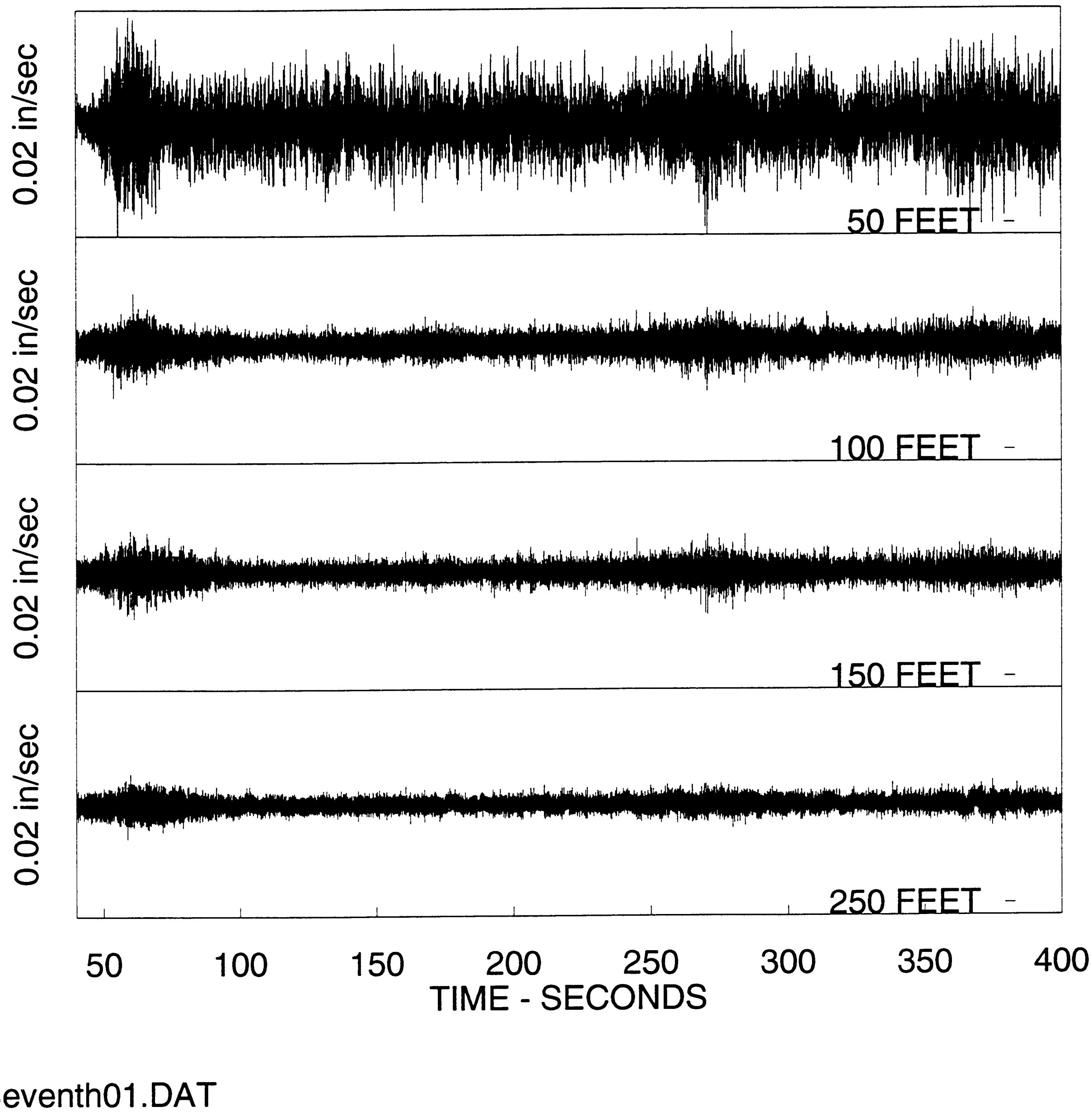


Figure 1 Particle Velocities during Freight Train Passage at Low Speed

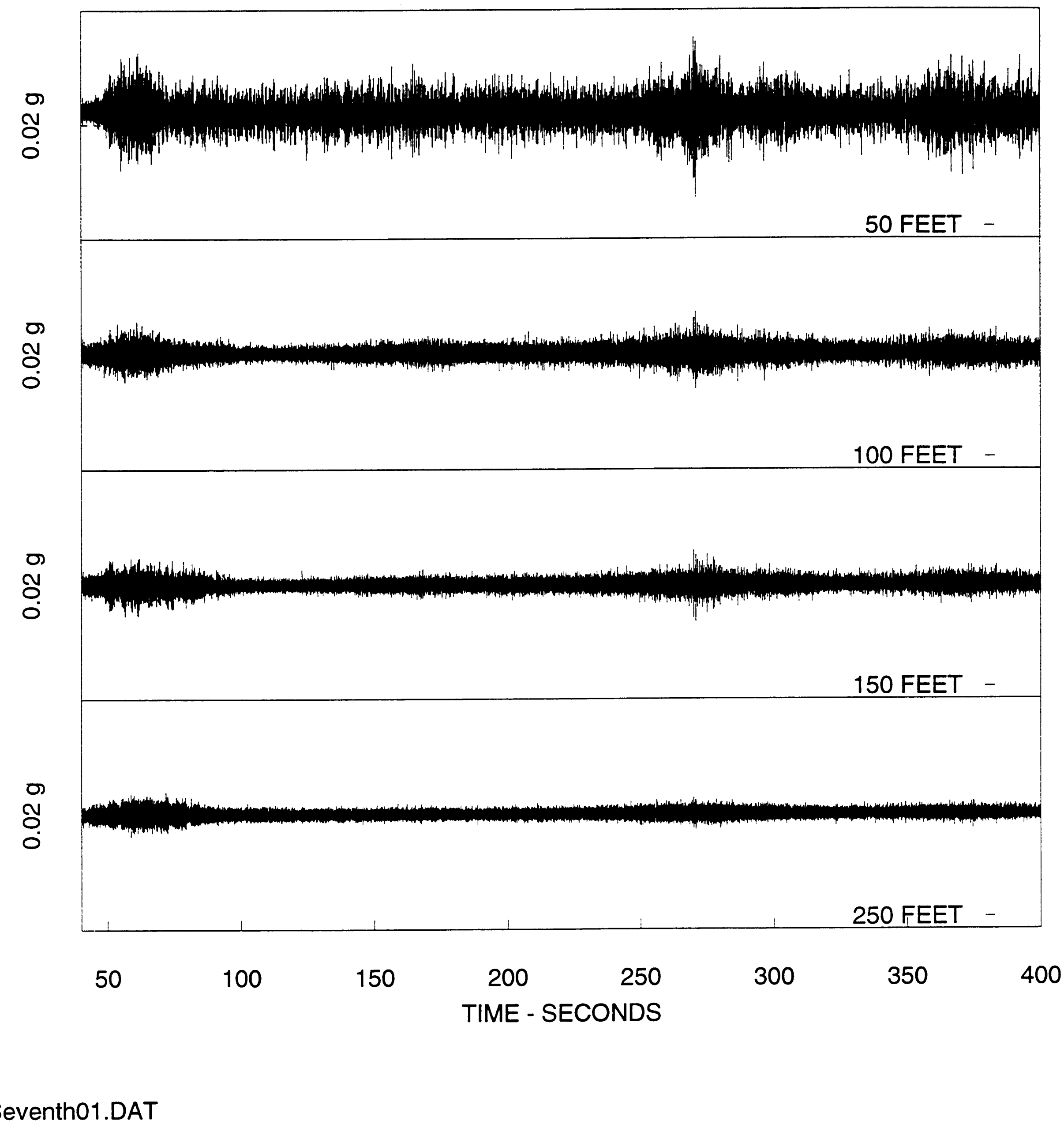


Figure 2 Ground Surface Particle Acceleration at Seventh Street During Feight Train Passage

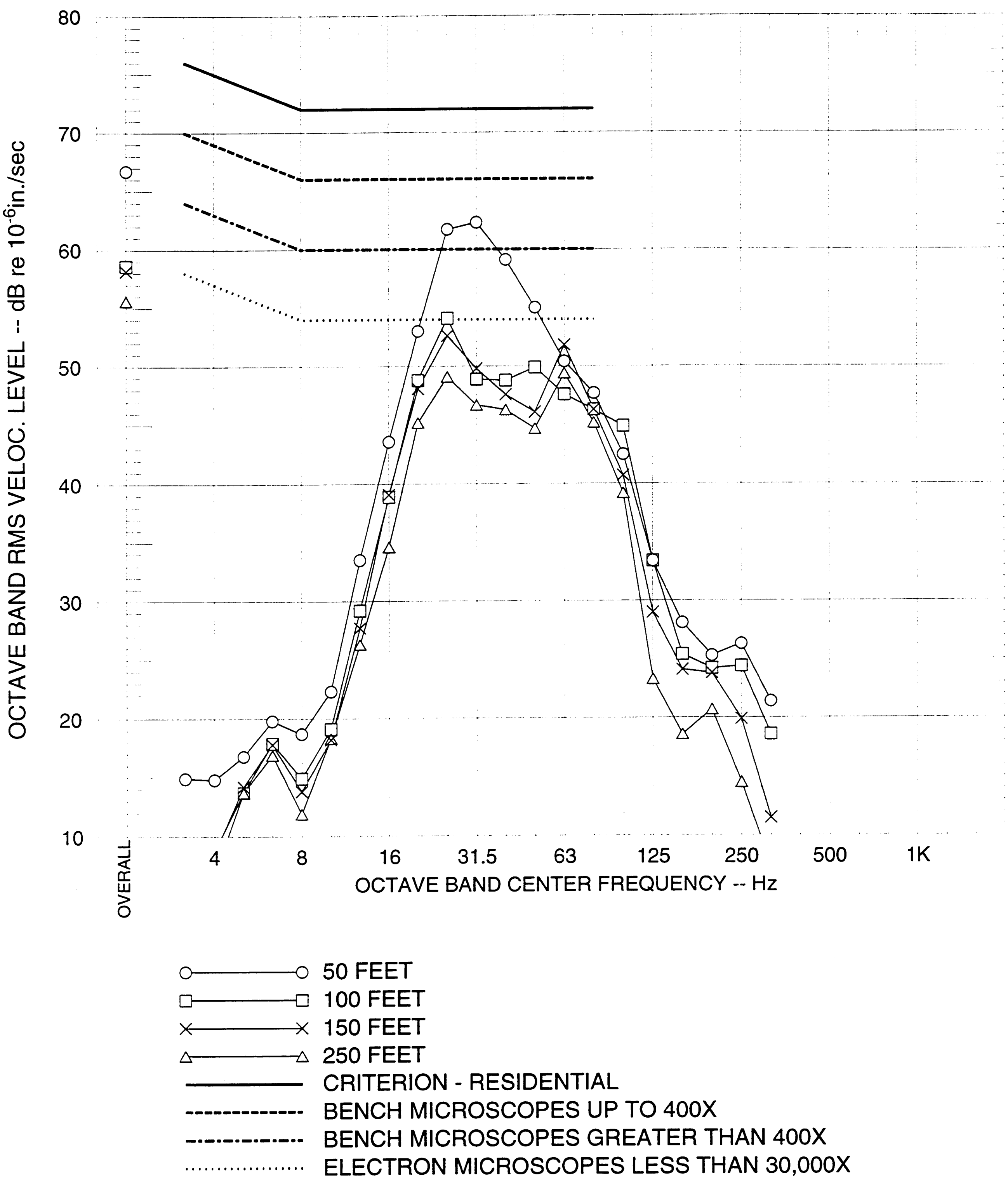


Figure 3 **Ground Vertical Vibration During Passage of Two Locomotives at 7th Avenue NW Grade Crossing (1st Sample)**

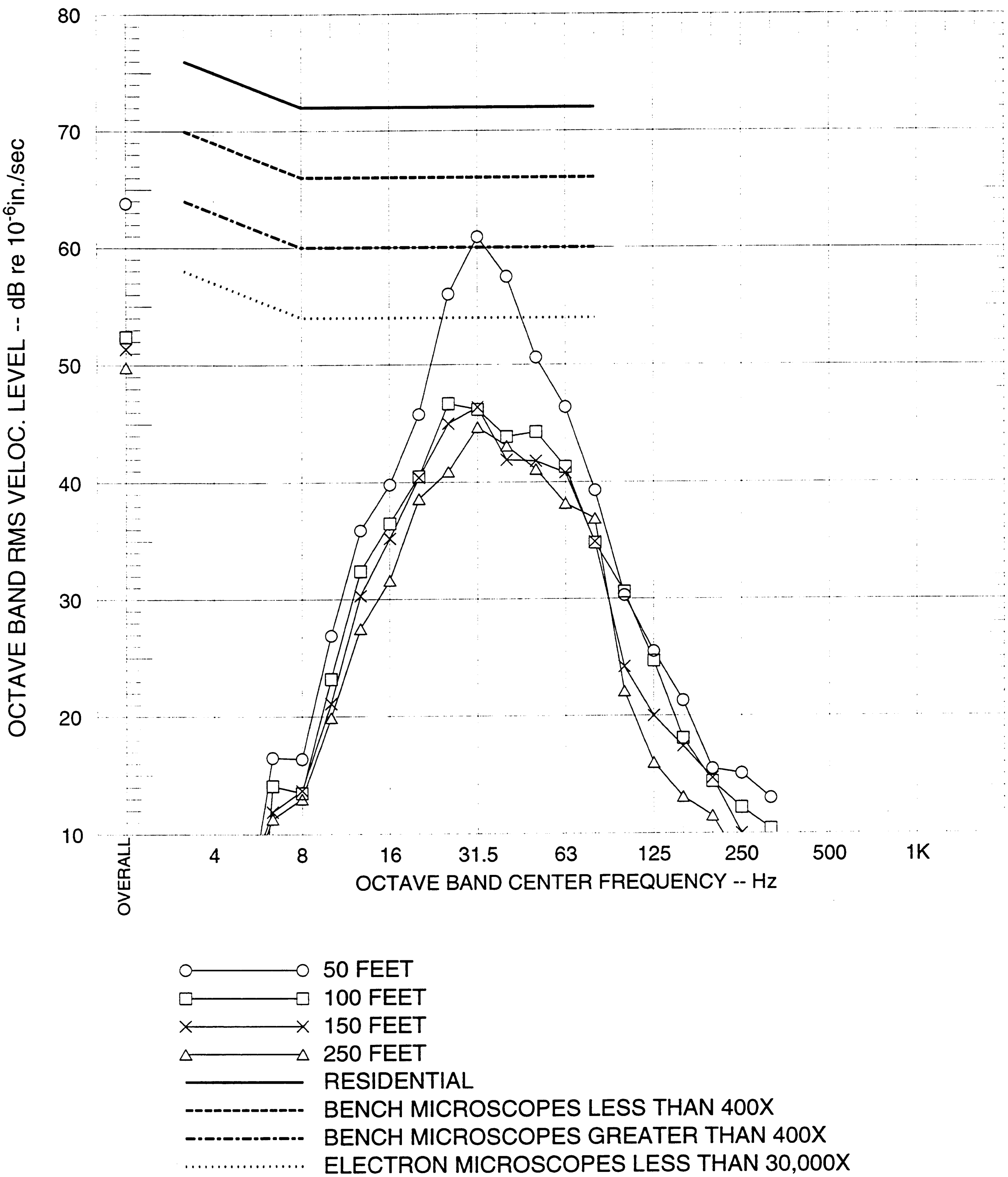


Figure 4 **Ground Vertical Vibration During Passage of Freight Cars at 7th Avenue NW Grade Crossing (2nd Sample)**

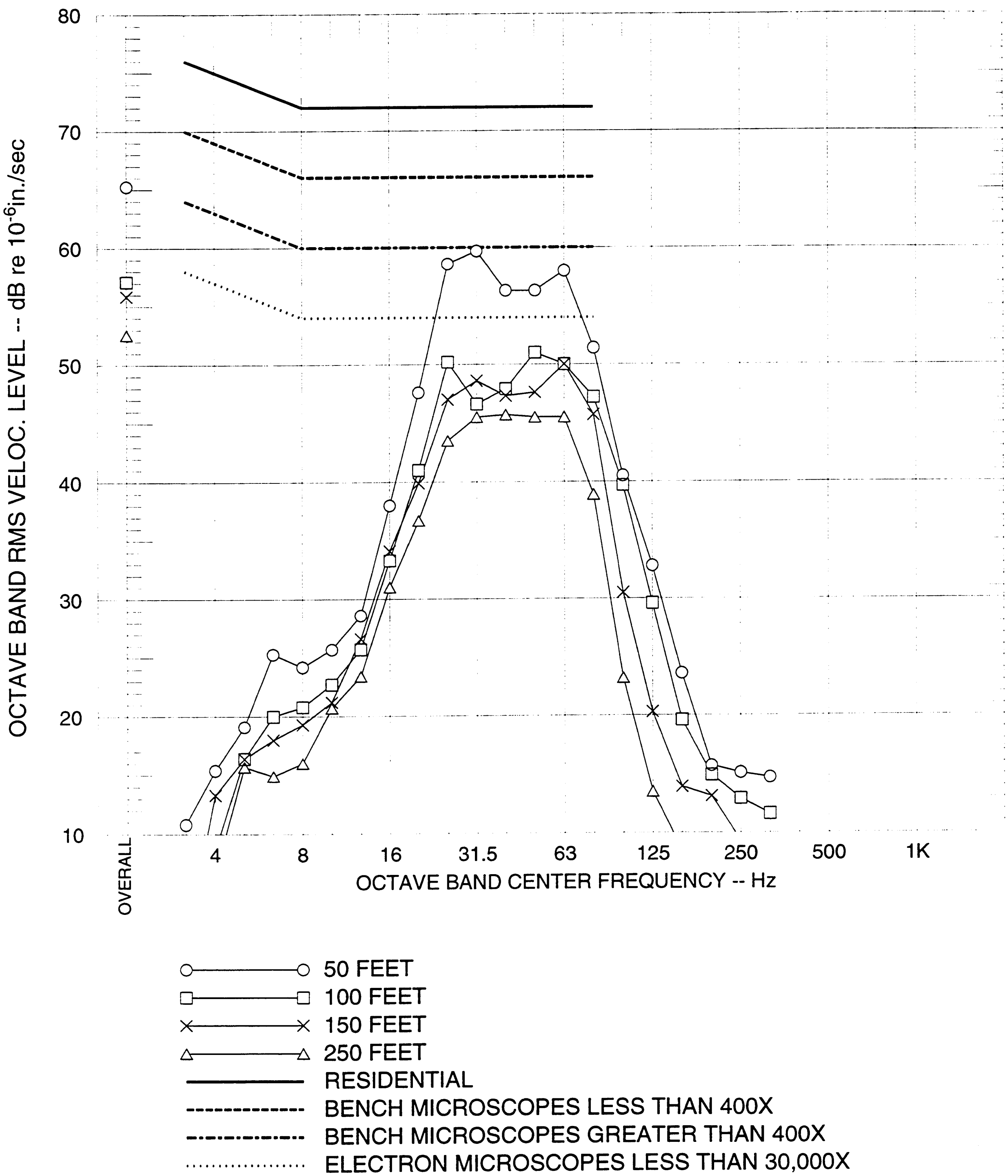


Figure 5 **Ground Vertical Vibration During Passage of Freight Cars at 7th Avenue NW Grade Crossing (3rd Sample)**

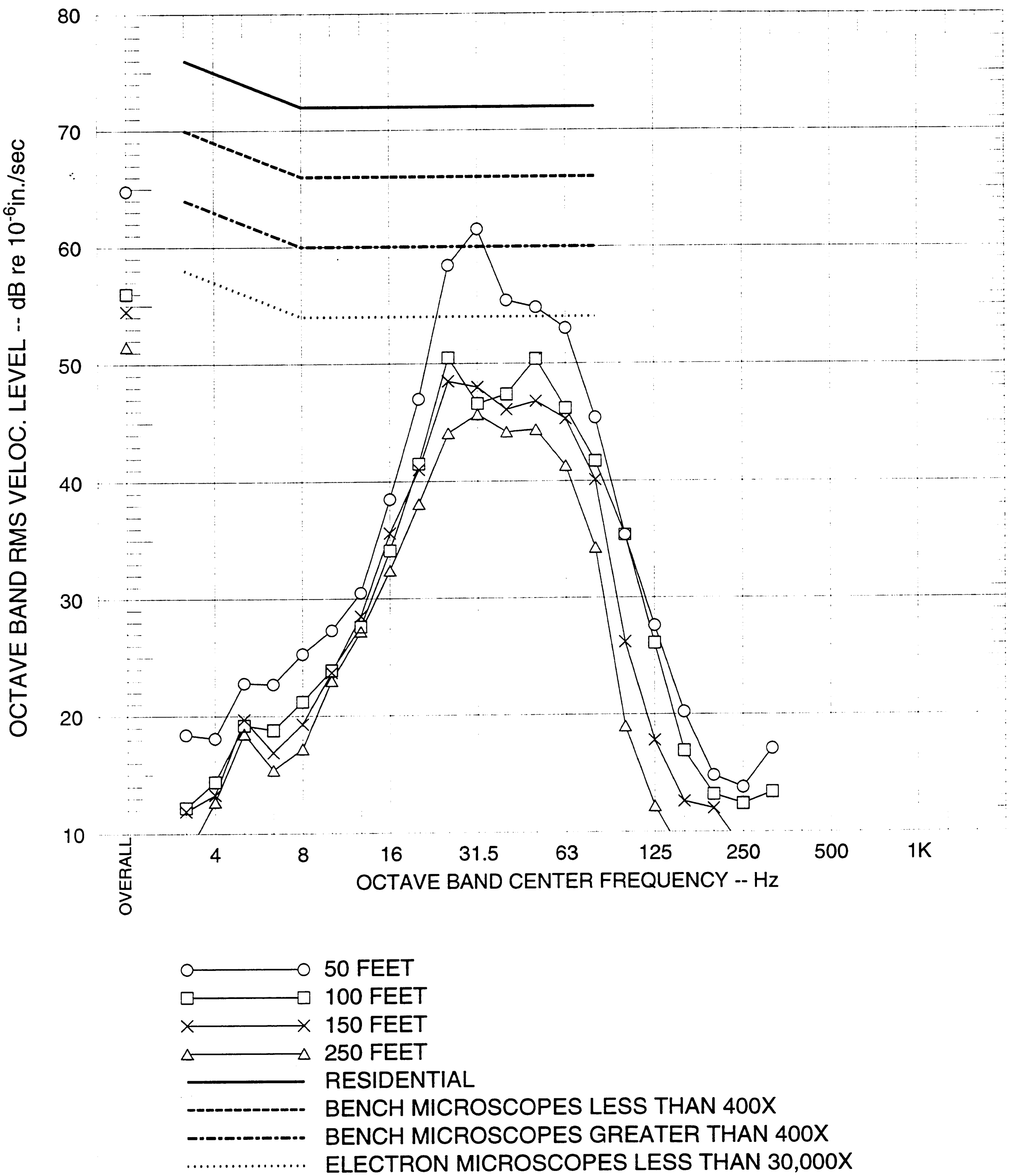


Figure 6 **Ground Vertical Vibration During Feight Passage at 7th Avenue NW Grade Crossing (4th Sample)**

Flood Wall Vibration in Mankato due to Railroad Trains

[THIS PAGE INTENTIONALLY LEFT BLANK]

WILSON, IHRIG & ASSOCIATES, INC.
ACOUSTICAL CONSULTANTS

5776 BROADWAY
OAKLAND, CA
U.S.A. 94618-1531
Tel: (510) 658-6719
Fax: (510) 652-4441
E-mail: info@wiai.com
Web: www.wiai.com

FLOOD WALL VIBRATION IN MANKATO DUE TO RAILROAD TRAINS

27 September 2001

Submitted to:

Mr. Stephen Thornhill
Burns & McDonnell
Kansas City, Missouri

Prepared by:

Dr. James T. Nelson
Wilson, Ihrig & Associates, Inc.
Oakland, California

INTRODUCTION

The DM&E Railroad proposes to upgrade track and introduce unit coal trains from the Powder River Basin through Mankato, Minnesota, using trackage rights. The upgrade would include addition of new track adjacent to existing track in the existing UP right-of-way, which follows a flood wall. Train speeds would be increased from the current maximum of 15 mph to approximately 45 mph. Concerns were raised by the City of Mankato regarding the impact of ground vibration on the flood wall due to the unit coal trains. Accordingly, flood wall vibration due to existing trains was measured. The data were compared with structural damage criteria, and were supplied to Burns and McDonnell for further evaluation with respect to liquefaction. The measurement results are presented here.

DAMAGE CRITERIA

No direct criteria for evaluating the flood wall vibration relative to damage were obtained. Criteria for building architectural damage due to long term ground vibration are typically of the order of 0.1 to 0.2 in/second ppv. Structural damage criteria for reinforced concrete buildings are usually of the order of 1 to 2 in/sec ppv, though a magnitude of 0.5 ppv might be considered for repetitive exposure. The nature of the wall and its foundation suggest that these criteria may not be directly applicable, though the criteria might provide an indication of the possible significance of the vibration. The tolerance of the wall and its foundation should be greater than building structures, as the wall's foundation is embedded in soil, and the wall is a solid reinforced concrete structure designed to hold back flood waters. Reinforced concrete retaining walls are often employed for retaining earth embankments supporting rail structures or as bridge abutments. The retaining walls in these cases are as little as fourteen feet from the track center. Surcharging (the lateral force acting against the wall due to vertical force on the soil) of the wall necessarily takes place during train passage, though the magnitude of this has not been determined. One may expect that for similar soil conditions, the surcharging and vibration of the flood wall would be comparable to or less than that experienced by concrete retaining walls and bridge abutments used for supporting railroad track structures.

There are other factors that may affect wall integrity, such as liquefaction of soils and residual settlement. Liquefaction has been addressed by Burns and McDonnell in another report, using the data obtained and reported here. The subject is discussed in a number of texts. (M. G. Spangler, R. L. Handy, *Soil Engineering, 4th Edition,* Harper & Row, New York, 1982.) Liquefaction can be caused by dynamically induced shear stresses in saturated cohesionless soils of low density and high void ratio (similar to high porosity). Earthquakes, heavy blasting, and pile driving may suddenly induce shearing stresses that tend to reduce the volume of the sand, which suddenly increases the hydrostatic excess pore pressure and decreases the intergranular pressure, causing a sudden decrease in intergranular shear strength. If the intergranular shear strength is reduced to below the applied shearing stress, the soil will fail in shear. Liquefaction usually occurs during earthquakes, when high acceleration of the order of the earth's gravitational acceleration (of the order of 1 g) occurs. However, liquefaction of soil could occur at lower accelerations, perhaps as low as 0.2 g. Additional study of this phenomenon would be needed to define appropriate vibration criteria for the flood wall relative to liquefaction.

Soil settlement over time can be a problem with heavily loaded foundations in non-cohesive soils. As with liquefaction, these settlements would be expected to be small unless the vibration acceleration were a significant fraction of 1 g. We have usually assumed that residual soil settlements from rail operations would not be significant if the ground acceleration were less than 0.1 g. However, this would apply to large or heavily loaded foundations, such as those supporting buildings or heavy machines. The inertial load on the flood wall foundation is relatively low, and the reaction forces produced by the foundation in response to a fraction of one gravitational acceleration (1 g) should be low compared to the static forces holding the foundation in place. Additional study of soil settlement should be conducted if the soils are non-cohesive and if the vibration acceleration exceeds this amount. A detailed discussion concerning residual soil settlements caused by ground vibration are described by Barkan (D. D. Barkan, *Dynamics of Bases and Foundations*, McGraw-Hill Book Co., New York, 1962).

MEASUREMENT PROCEEDURE

Seismic accelerometers were adhered to the floodwall base at two locations along the wall. One location is designated as Location A, and the other Location B. The two locations were separated by approximately 100 feet. The wall was located at about 37 feet from the near track center.

Two accelerometers were employed at each location. One was oriented in the vertical direction, and the other in the horizontal direction, transverse to the wall and track. The horizontal component parallel with the track was not measured, because this was not considered to be as significant as the transverse and vertical directions, and certainly should be of lesser magnitude.

The laboratory analyses included reproduction of time domain waveforms to determine peak accelerations and velocities. This process involved reproduction of the digital data, low pass filtering, decimation, and transfer to computer disk. The acceleration data were then integrated numerically by convolution with a "simple RC" filter response function.

RESULTS

The results are presented for both vibration velocity and vibration acceleration. For each of these, instantaneous magnitudes are presented. The instantaneous magnitude is the actual time dependent variation of the acceleration or velocity, and may be positive or negative, with average value zero.

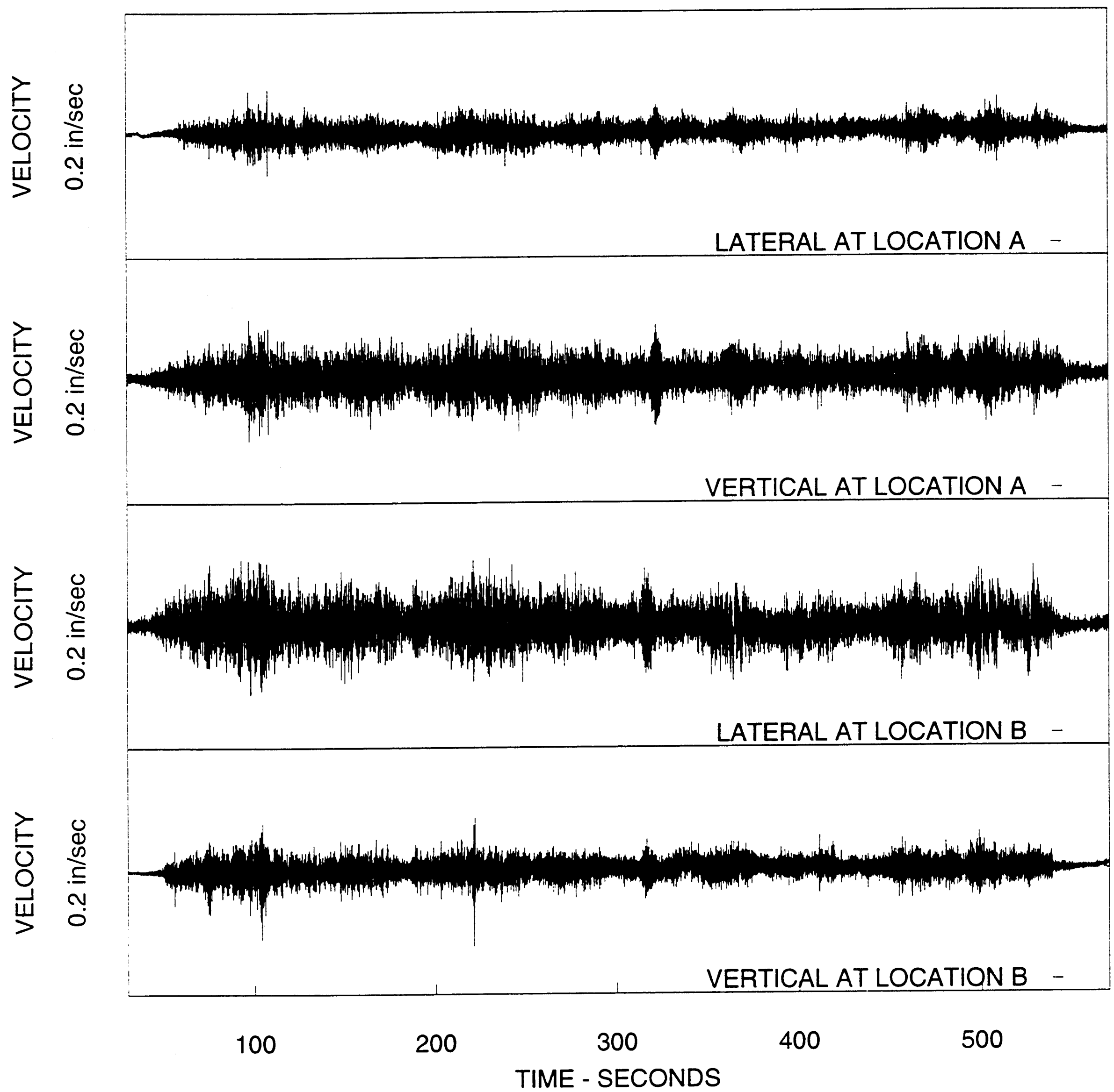
Velocity

Flood wall vibration velocity versus time are presented in Figure 1 for all four accelerometers. The scales at the left-hand side refer to the span of the chart. In this case, the span is 0.2 inch/sec, so that the positive and negative full scales are plus and minus 0.01 in/sec, respectively. The entire 9 minutes of recorded velocity data obtained during train passage are represented for a bandwidth of approximately 1 Hz to 100 Hz. Throughout the entire train passage, the magnitude of the vibration was less than 0.05 in/second. At Location A, the vertical component was greater than the lateral component. However, at Location B, the lateral component was greater than the vertical component.

Most of the vibration is of relatively low magnitude, with infrequent transients that approached 0.05 in/sec.

Figure 2 illustrates the waveform of the transient velocity occurring between 100 and 110 seconds into the sample. The peak velocities were of the order of 0.05 in/sec. The period of the velocity transient appears to be about 10 to 11 Hz. This sample is representative of other transients occurring during the sample.

Anomalous transients occur at about 565 seconds and 535 seconds into the data on Channel 3. Figure 3 and Figure 4 are temporal expansions of these respective transients. The waveform contains long period dips that are believed to be due to thermal transients and should be discounted. The transients would likely be due to the wind blowing against the accelerometer, which produced thermal effects under certain circumstances. If these were real data, the transient would also have appeared in the vertical data at Location B. This problem normally does not occur in this type of measurement, so the result is surprising.



$0

Figure 1 **Vibration Velocity at Flood Wall During Train Passage**

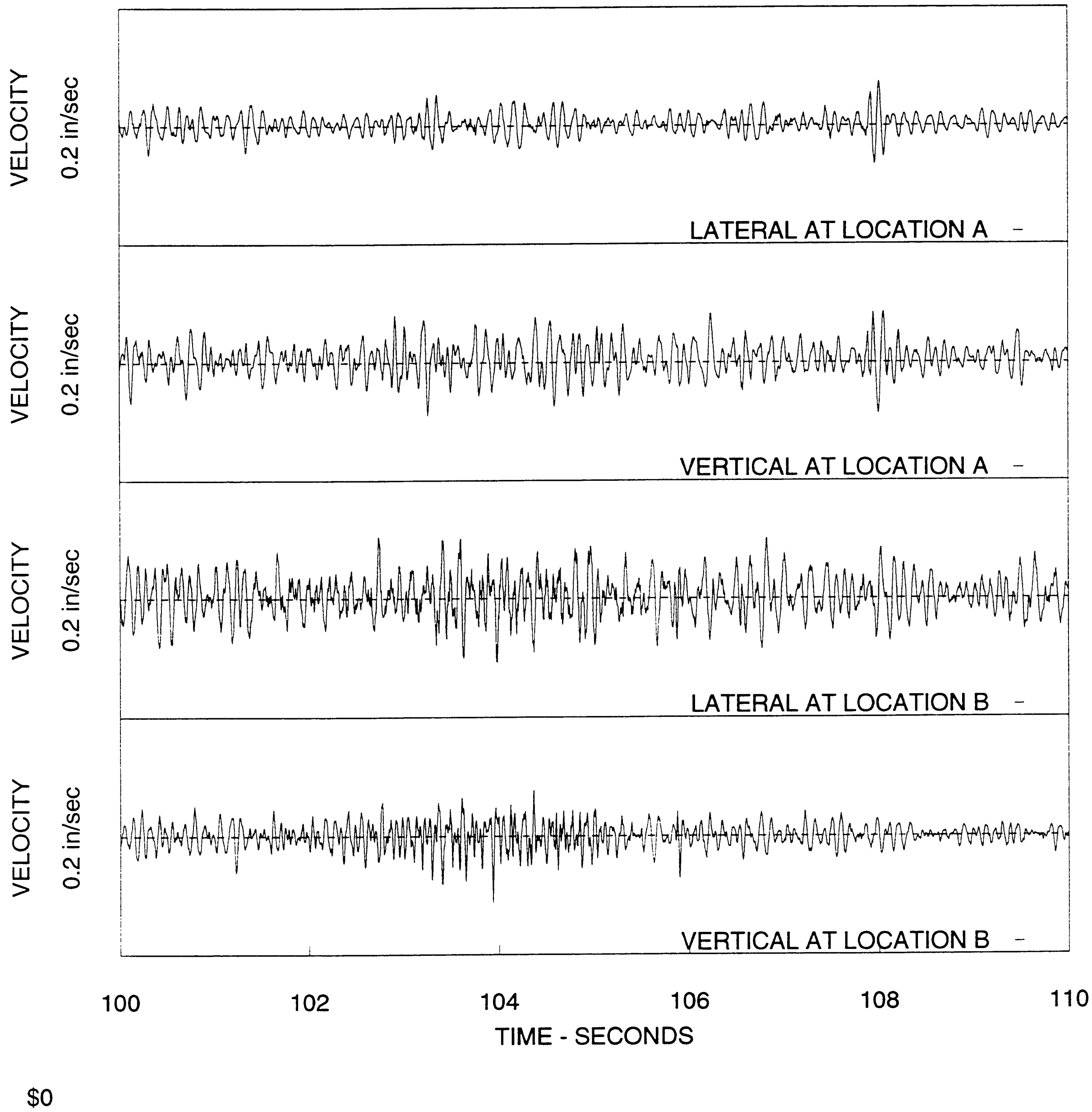
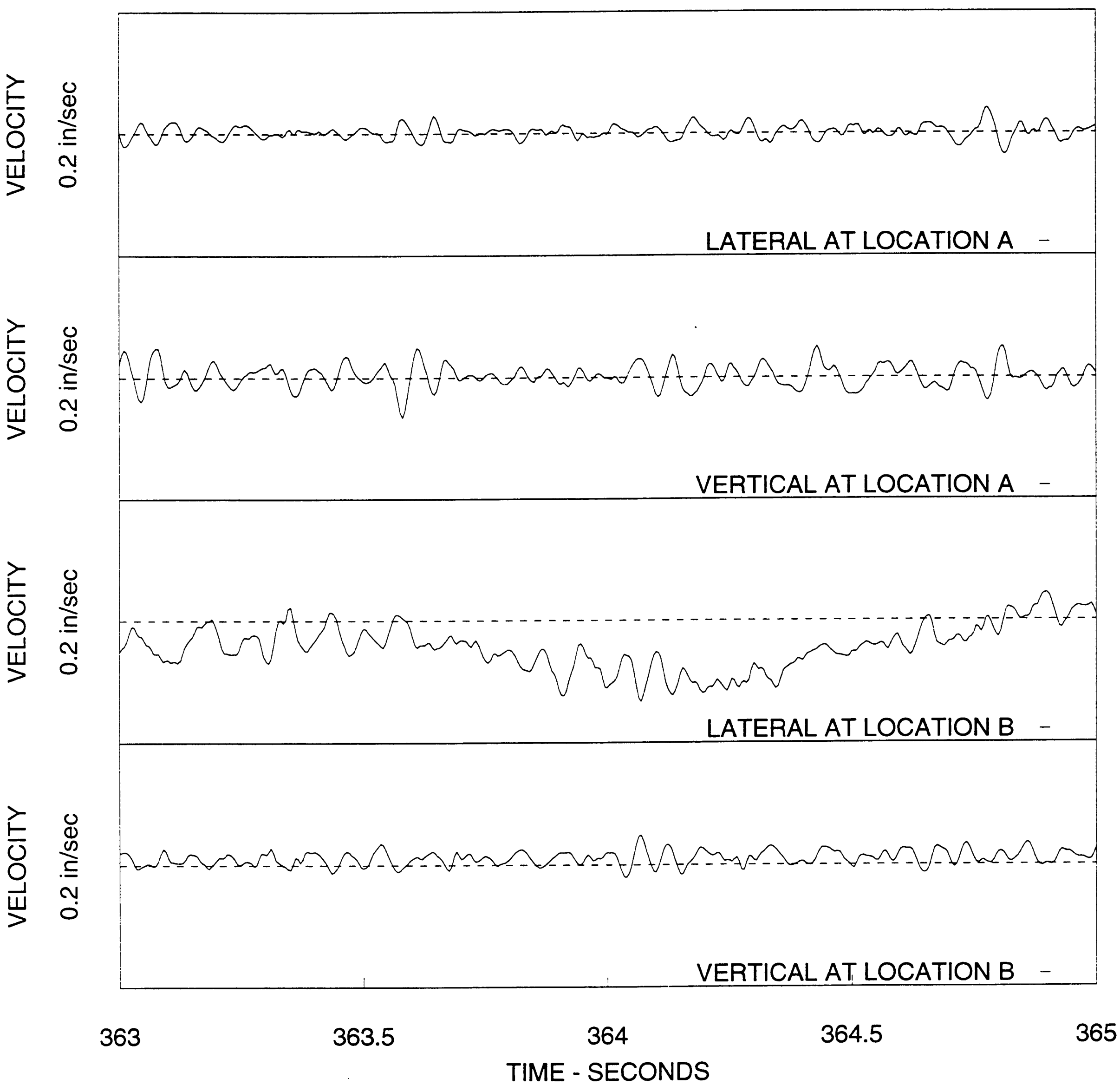


Figure 2 **Transient Vibration Velocity at Flood Wall During Train Passage**



$0

Figure 3 **Anomolous Low Frequency Transient at 364 Seconds Attributed to Low Frequency Thermal Drift on Channel 3**

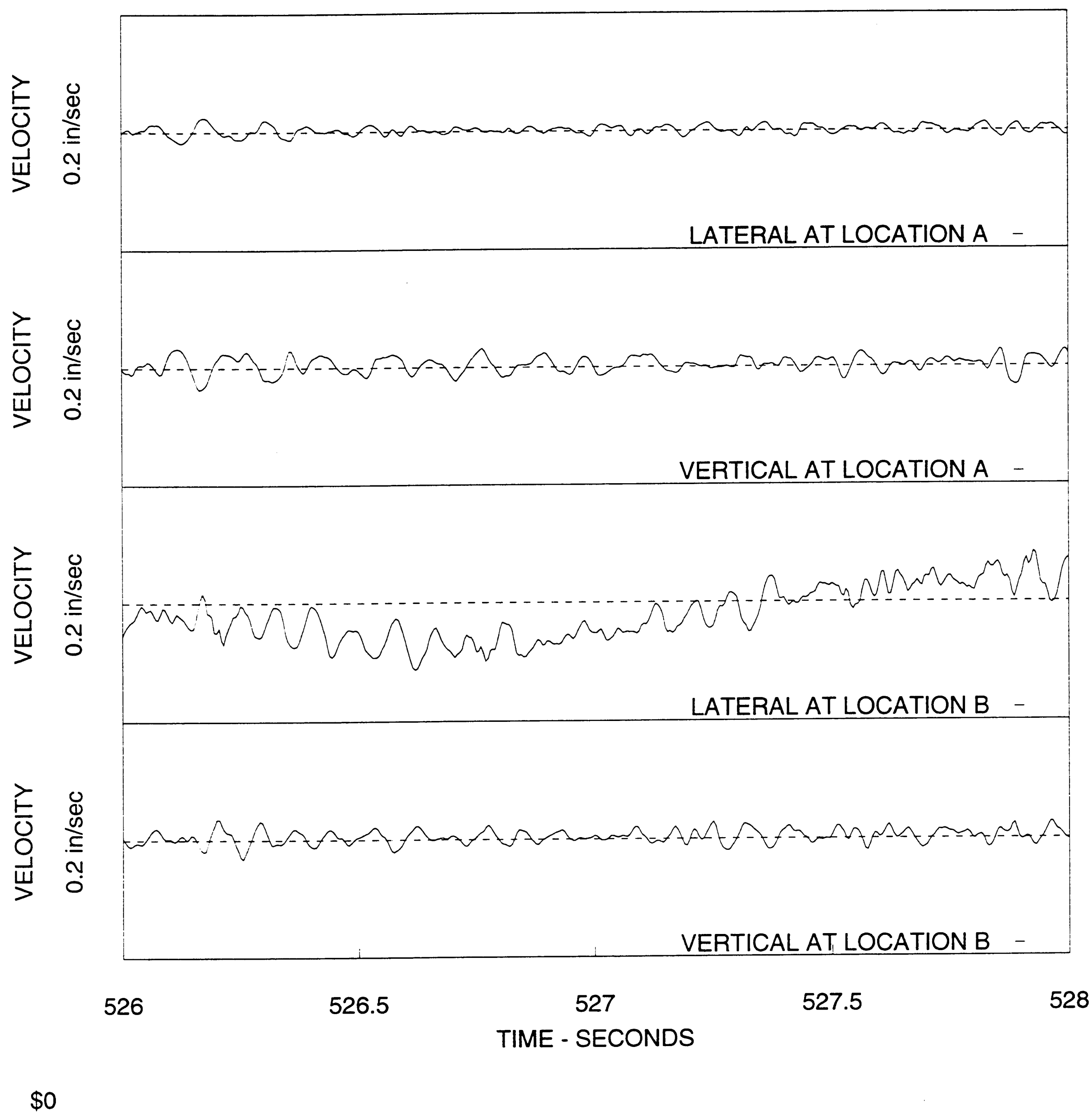
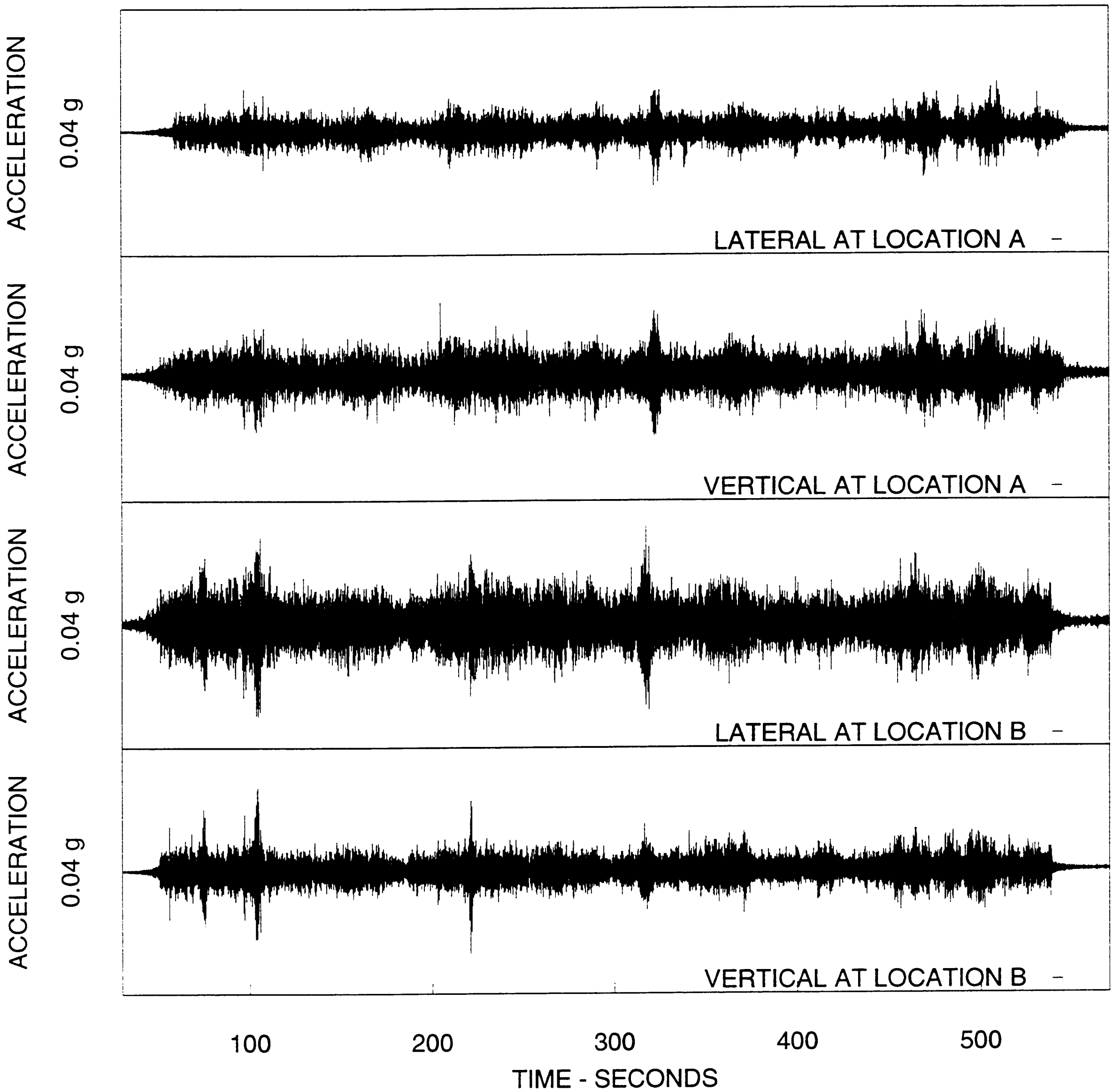


Figure 4 **Anomolous Low Frequency Transient at 535 Seconds Attributed to Low Frequency Thermal Drift on Channel 3**

Acceleration

Figure 5 illustrates the acceleration magnitudes corresponding to the velocity data presented in Figure 1. The spans of the plots are 0.04 g, providing a range of plus and minus 0.02 g. The typical magnitude of acceleration was of the order of 0.005 g, with transients approaching or slightly exceeding plus or minus 0.01 g.

The acceleration transient occurring between 100 and 110 seconds into the sample was expanded and plotted in Figure 6. This sample corresponds to the velocity data presented above in Figure 2.



$0

Figure 5 Vibration Acceleration at Flood Wall During Train Passage

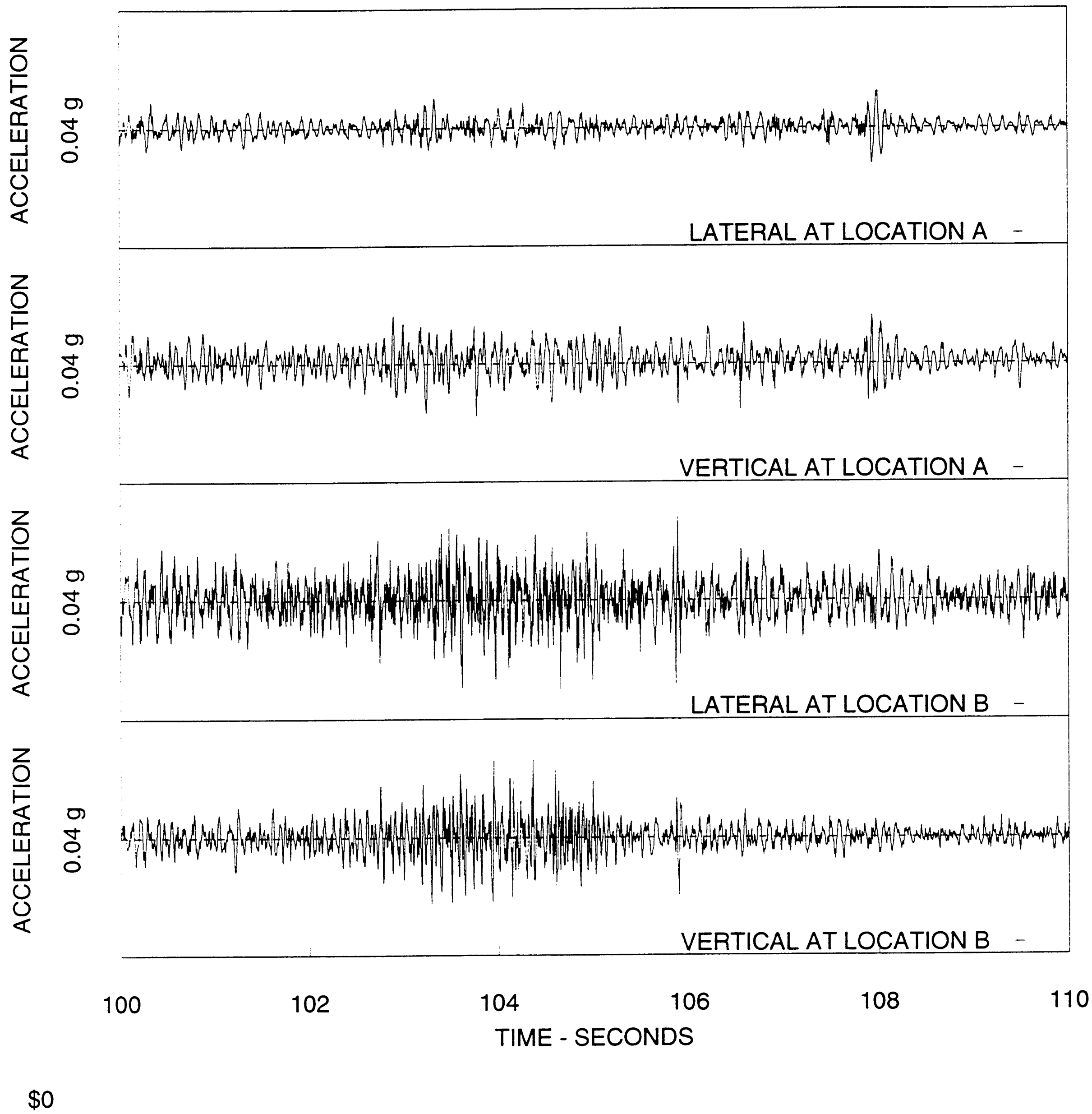


Figure 6 Particle Velocity Transients During Train Passage

DISCUSSION

The maximum vibration velocity is well below structural damage criteria that might be applied to reinforced concrete structures. The maximum acceleration magnitude of roughly 0.01 g is approximately 1/100th the magnitude that might be associated with a major earthquake and which might be considered to lead to liquefaction of non-cohesive soils under high dynamic loading, and 1/10th of that which might be considered to pose a problem for settlement under cyclic dynamic forces. At least for these train data, the vibration levels would appear to be acceptable.

Introduction of new track at roughly 18 feet from the floodwall and nominally 45 mph trains would result in higher magnitudes of vibration. The increase is difficult to predict. A review of Bousinesq's theory for point loads on a half space, suggest that the reduced distance would produce an increase of roughly 100%, because the vibration response of the soil surface will likely double for each having of distance from the source. Line source effects would likely not apply, because the vibration observed at the flood wall would be controlled by the nearest truck during passage. The train would probably appear as a line source when the track is at roughly 50 feet from the wall, that is, at a distance comparable with the truck separation of the cars. The peak velocity during locomotive passage, illustrated in Figure 2, would probably be increased to about 0.1 in/second from about 0.05 in/second. Again, this would be well within structural damage criteria.

Train speed increases would also tend to increase the vibration velocity of the wall. The usual assumption is that the vibration energy would increase linearly with train kinetic energy. That is, the vibration magnitude would double with each doubling of train speed. Assuming that train speeds might be increased to 45 mph would suggest that the wall vibration amplitude might be increased by about a factor of 4.5, giving a vibration velocity amplitude of 0.45 in/second, assuming that track conditions are the same. However, continuous welded rail will be employed, and there would likely be an improvement in ballast conditions, so that some reduction of vibration amplitude might be expected. The peak vibration magnitudes would likely be less than 0.5 in/second with the unit trains running at 45 mph on track with center at about 18 feet from the wall. This magnitude is still within typical structural damage criteria for reinforced concrete structures.

With respect to acceleration, similar arguments can be employed to suggest that the acceleration magnitude would increase by roughly 2 due to track location, and 4.5 due to speed increase. The existing magnitudes of 0.015 g could be increased to approximately 0.135 g. This magnitude is could be sufficient to produce liquefaction or dynamic residual settlement of heavily loaded foundations in granular soils, though it is at the extreme low end of the range of concern, and should be of concern only if the soil were fully saturated to the surface, cohesionless, and of low density, or high void ratio.

Information concerning settlement of lightly loaded sheet pile supported foundations has not been obtained. We would expect that lightly loaded sheet pile foundations would be much more tolerant to ground vibration than heavily loaded spread-footing foundations with respect to residual settlement.

Mitigation

Peak acceleration magnitudes of the order of 0.135 g might cause soil settlement over time if the soils are non-cohesive sands. However, if this were to be a problem, there would also be a problem with the soil settlement beneath the railroad trackbed. If this is judged to be significant, a possible mitigation measure might include pressure grouting of the soil beneath the track.

According to Spangler and Handy, (*Soil Engineerign, pg. 665),* liquefaction occurs only once for a particular maximum acceleration in a particular deposit, since after liquefaction the sand particles settle into a denser, more tightly packed, matrix. The phenomenon is limited to relatively young deposits such as might be found in river deltas or alluvial sands. Liquefaction potential can be reduced by vibratory compaction or other vibratory means. These would induce settlement, and may cause some settlement of the wall. In this case, pressure grouting or injection of chemical solidifiers can be considered.

A geotechnical engineer should be consulted concerning the need for soil stabilization, grouting procedures, depth, and lateral extent. Pressure grouting is used by railroads for stabilizing soft soils, and railroad equipment should be readily available for performing this soil stabilization process.

[THIS PAGE INTENTIONALLY LEFT BLANK]